MODELLING OF SHEAR STRENGTH OF CONCRETE BEAMS WITH STEEL AND GFRP REINFORCEMENT

Jakub VEČEŘE[1], *Drahomír NOVÁK*[1], *Renata KOTYNIA*[2], *Damian SZCZECH*[2], *Monika KASZUBSKA*[2]

[1]Institute of Structural Mechanics, Faculty of Civil Engineering, Brno University of Technology, Veveří 95, 60200 Brno, Czech Republic
[2]Lodz University of Technology, Al. Politechniki 6, 90-924 Lodz, Poland

jakub.vecere@seznam.cz, novak.d@fce.vutbr.cz, renata.kotynia@p.lodz.pl, damian.szczech@p.lodz.pl, monika.kaszubska@p.lodz.pl

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Abstract. *Extensive experimental programme has been performed recently at Lodz University of Technology, Poland, focused on shear failure testing of concrete beams. One of the goals of the research was to develop an understanding of the differences between steel and composite reinforcement impact on shear strength of concrete beams. The aim of this paper is to show aspects and capabilities of numerical modelling of two selected beams reinforced by steel and glass fiber reinforced polymer. Modelling is performed by nonlinear fracture mechanics finite element. Modelling is focused also on bond between concrete and reinforcement, the importance of correct bond modelling is extremally important for GFRP reinforcement.*

Keywords

Shear strength, nonlinear modelling, fracture mechanics, composite reinforcement, bond

1. Introduction

The shear strength of concrete beams has been extensively studied over the last five decades. Many reinforced concrete (RC) beams produced in 1960s were designed without shear reinforcement. Since a significant portion of the beams are still in use without problems, a reasonable question that needs to be clarified is the reliability of existing codes for prediction of the shear capacity of RC members without transverse reinforcement. The intense research was carried out, by investigating the influence of the following parameters on the shear capacity of this type of structural members: size effect, concrete strength, shear span to depth ratio, and flexural reinforcement. Despite the strong research effort on the shear transfer mechanisms in RC beams without stirrups this issue still raises many controversial opinions due to difficulties of isolating each mechanism and capturing its influence on the shear capacity, as well as the large scatter of results [1].

The aim of this research is to investigate the shear failure mechanisms in T-shape, single span and simply supported beams reinforced with longitudinal steel or glass fiber reinforced polymer (GFRP) bars. Usually, the critical shear crack in RC beams without stirrups develops through the theoretical compression strut preventing a direct transfer of the shear force to the support. The main parameter affecting the crack pattern and the shear strength of the beams is the shear slenderness. Moreover, key parameters that influence bond performance of GFRP bars to concrete are different than these of steel bars, so the bond behaviour of GFRP bars to concrete is expected to vary from that of conventional steel reinforcement. The modulus of elasticity and the shear stiffness of GFRP bars are lower than steel. Moreover, the GFRP coefficient of thermal expansion is different from that of steel or concrete (CEB-fib, 2000). In fact, the experimental tests carried out with concrete beams longitudinally reinforced with GFRP bars and without transverse reinforcement indicated that when using this type of reinforcement instead of steel bars, the shear strength of the beams is smaller [2], [3].

2. Shear tests

Experimental tests has been performed at the Laboratory of Concrete Structures, Lodz University of Technology, based on shear tests of the beams with the following investigated parameters, [4]: cross section: rectangular (R) and T-section (T); type of concrete: normal concrete (ready-mixed concrete) with various compressive strength; variable longitudinal reinforcement: steel (S) and glass fiber reinforced polymer (GFRP); variable longitudinal reinforcement ratio; variable shear reinforcement: beams without steel transversal reinforcement (NT) and beams with steel transversal reinforcement (T); variable concrete cover thickness. Research program of GFRP and steel reinforced beams with two compressive strength contains 33 beams (18 beams C25/30 and 15 beams: C30/35) [4].

Configuration set-up is shown in Fig.1 and sensor system deformation measurement is in Fig. 2.

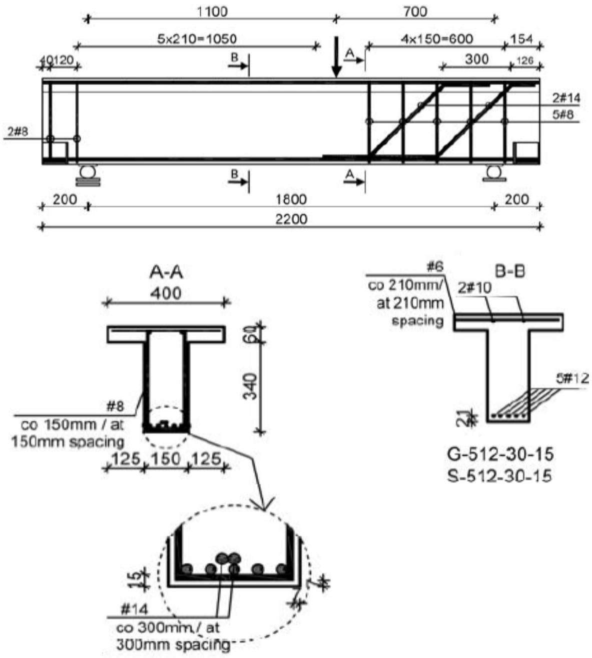

Fig. 1: Configuration of reinforced beams (adopted from [4]).

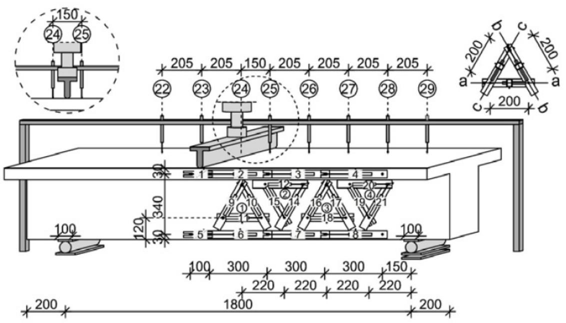

Fig. 2: Sensors placement for deflection monitoring (adopted from [4]).

3. Nonlinear fracture mechanics finite element computational models

The nonlinear finite element analysis (NLFA), implemented in software ATENA enables realistic modelling of the damage processes in the structural material [5], which is essential for shear failure modelling. It is based on elite nonlinear material models allowing realistic analysis of the structural response, damage and post-peak behavior. Nonlinear material models for concrete, steel reinforcement and bond are available. Such modelling belongs to a category of high-fidelity computational modelling. In the models for concrete the tensile cracking is an important phenomenon. Concrete in tension is handled by the nonlinear fracture mechanics based on fracture energy; the approach utilizes the orthotropic smeared crack formulation and the crack band theory. It employs the Rankine failure criterion, exponential softening, and it can be used as a rotated or a fixed crack model. For concrete in compression the plasticity model based on the Menétrey–Willam failure surface according to [6] is adopted. The plastic volumetric strain as a hardening/softening parameter and a non-associated flow rule based on a nonlinear plastic potential function are utilized. The model can be used to simulate concrete cracking, crushing under high confinement and crack closure due to crushing in other material directions. Software ATENA has been used successfully for prediction of shear strength of large concrete beams [7].

For the steel reinforcement elastic–plastic behavior is assumed. The parameters of the material models include fracture–mechanical parameters of concrete, such as tensile strengths, fracture energy, bond strengths, etc.

2D computational model has been developed for our modelling, Fig. 3. Beams were simulated by parameter analysis of Arc Length. Following material parameters has been used:

T-beam with steel reinforcement: material of concrete 3D Non Linear Cementitious 2; compressive strength of concrete 31.1 MPa; cubic compressive strength of concrete: 33.2 MPa; modulus of elasticity 273 333 MPa; fracture energy $6.198 \cdot 10^{-5}$ MN·m^{-1}; concrete tensile strength 2.9 MPa; yield strength 526 MPa; tensile strength 638 MPa; modulus of elasticity 198 300 MPa.

T-beam with GFRP reinforcement: material of concrete 3D Non Linear Cementitious 2; compressive strength of concrete 30.1 MPa; cubic compressive strength of concrete: 31.6 MPa; modulus of elasticity 259 000 MPa; fracture energy $5.997 \cdot 10^{-5}$ MN·m^{-1}; concrete tensile strength 2.8 MPa; tensile strength 1 195 MPa; modulus of elasticity 50 200 MPa.

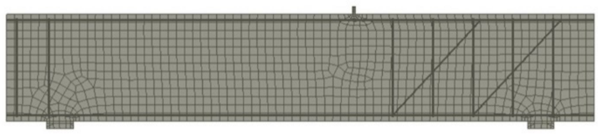

Fig. 3: Computational mode in ATENA 2024.

4. Bond modelling

The interaction of concrete and reinforcement through the bond and its understanding is a crucial problem when modelling the crack propagation in reinforced concrete.

Most engineering approaches describe the bond by a bond-slip relationship. However, such model is not generally applicable due to the difficulty in slip definition and boundary effects. Some studies indicated that the fracture-based model of the interface damage is more objective [7]. The importance of correct bond modelling is extremally important for GFRP reinforcement.

In ATENA there are three pre-defined reinforcement bond model for concrete-steel (Bond model by Bigaj, CEB-FIP 1990 and CEB-FIP 2010 model code), but none for concrete-GFRP. Since we did not have the measured data of slip-bond relationship of concrete-GFRP, from which is possible to model bond by ATENA-GiD, we had to estimate it. Bond models that we used are Bigaj and CEB-FIP 1990.

The best results we achieved so far involve reduced bond strength by 50 % compared to the bond strength for concrete-steel. The bond only for lower reinforcement was considered.

4.1. Bond model by Bigaj

The bond model based on the work [8] is one of the pre-defined models of bond in software ATENA [9]. The slip law for this model is shown in Fig. 4. Bond by Bigaj is defined only for ribbed bars [9].

The input values that are needed include these two: concrete cubic compressive strength $f_{c,cub}$ (MPa) and reinforcement bar diameter d (m). Other parameter is quality of bond, which can be excellent, good or bad [9].

The four points, which defines bond-slip law by Bigaj, are defined by these following equations:

$$s' = \frac{s}{d}, \tag{1}$$

$$\tau'_b = \frac{\tau_b}{\sqrt{0.8\, f'_{cu}}}, \text{for } f'_{cu} < 60\ MPa,$$

$$\tau'_b = \frac{\tau_b}{\sqrt{0.88\, f'_{cu}}}, \text{for } f'_{cu} > 60\ MPa, \tag{2}$$

where s is slip value in (m), d is reinforcement bar diameter in (m) and s' is pre-defined constant for each point included in [9], τ_b is bond strength in (MPa), f'_{cu} is concrete cubic compressive strength in (MPa) and τ'_b is pre-defined constant for each point included in [9].

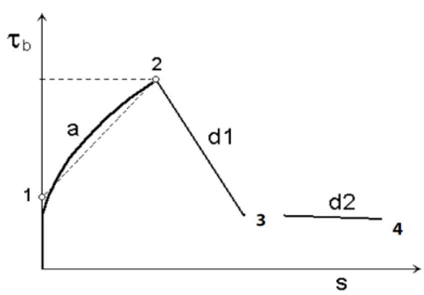

Fig. 4: Bond-slip law by Bigaj 1999 (adopted by [9]).

4.2. CEB-FIP 1990 model code

The CEB-FIP 1990 [10] model code is other pre-defined model in software ATENA. The bond-slip law is shown in Fig. 5. The CEB-FIP can be used for ribbed or smooth bars.

The input values that are needed include these two: concrete compressive strength f_c (MPa) and reinforcement bar diameter d (m). Other parameter is quality of bond, which can be good or all other cases [9]. Then we can choose between Unconfined concrete (Failure by splitting of concrete) or Confined concrete (Failure by shearing of the concrete between ribs).

The CEB-FIP 1990 [10] model code is defined by these equations:

$$\tau_b = \tau_{max}\left(\frac{s}{s_1}\right)^{\alpha}, 0 \le s \le s_1, \tag{3}$$

$$\tau_b = \tau_{max}, s_1 < s \le s_2, \tag{4}$$

$$\tau_b = \tau_{max} - \left(\tau_{max} - \tau_f\right)\left(\frac{s - s_2}{s_3 - s_2}\right), s_2 < s \le s_3, \tag{5}$$

$$\tau_b = \tau_f, s_3 < s, \tag{6}$$

where τ_b is bond strength in (MPa), s is slip value in (m), τ_{max} is maximum bond strength in (MPa), and τ_f is bond strength, where linear curve passes to non-linear curve in (MPa).

Slip value s and constant α depends on the bond quality and type of failure [9]. For ribbed bars in unconfined concrete these following equations apply:

$$\tau_{max} = 2.0\sqrt{f_c}, \text{for good quality,} \tag{7}$$

$$\tau_{max} = 1.0\sqrt{f_c}, \text{for all other cases,} \tag{8}$$

$$\tau_f = 0.15\tau_{max}, \text{for all cases,} \tag{9}$$

For ribbed bars in confined concrete these following equations apply:

$$\tau_{max} = 2.5\sqrt{f_c}, \text{for good quality,} \tag{10}$$

$$\tau_{max} = 1.25\sqrt{f_c}, \text{for all other cases,} \tag{11}$$

$$\tau_f = 0.4\tau_{max}, \text{for all cases,} \tag{12}$$

where τ_{max} is maximum bond strength in (MPa), f_c is concrete compressive strength in (MPa) and τ_f is bond strength, where linear curve passes to non-linear curve.

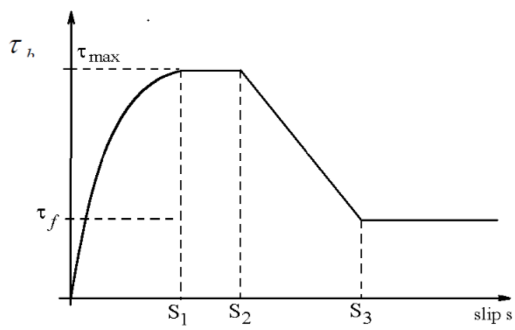

Fig. 5: The bond-slip law by CEB-FIP 1990 (adopted by [9]).

5. Results

Simulation of shear strength have been performed for many alternatives of material parameters and bond assumption. Here are the best results achieved so far.

5.1. Beam reinforced by steel

We assumed that bond-slip relationship would not play such a large role in beam reinforced by steel. However, the load-deflection curve of CEB-FIP 1990 model code and load-deflection curve of the specimen for which we have not modelled bond were different.

We use 2 deflection monitors (v25, v26) to compare data computed by ATENA with data that we receive from Laboratory of Concrete Structures, Lodz University of Technology.

1) Bond by Bigaj

Crack patterns are compared with experiment in Fig. 6.

The load-deflection curves of Bigaj are shown in Fig. 7 and Fig. 8. Material parameters were adopted from experimental research. Values that were achieved at maximal load force are:

Experiment: maximum load 142.95 kN; deflection at monitoring point v25 1.82 mm; deflection at monitoring point v26 1.608 mm.

ATENA: maximum load 162.08 kN; deflection at monitoring point v25 1.88 mm; deflection at monitoring point v26 1.65 mm.

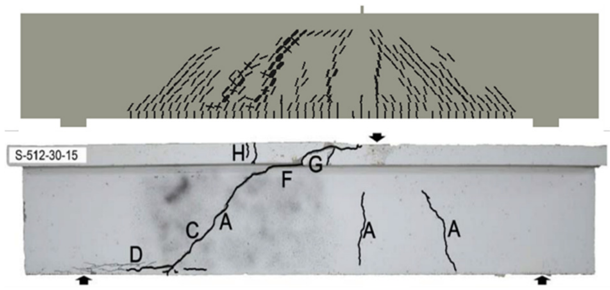

Fig. 6: Crack patterns from experiment and modelling (Bigaj).

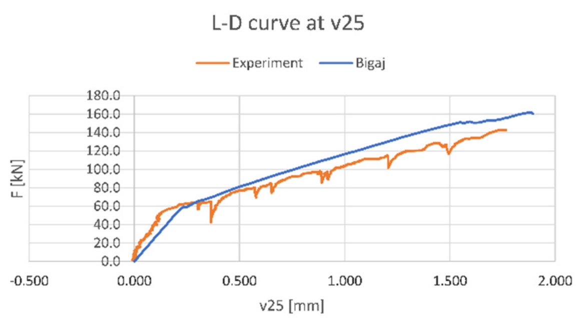

Fig. 7: Load-deflection curve – experiment vs. results of modelling, deflection at monitoring point v25.

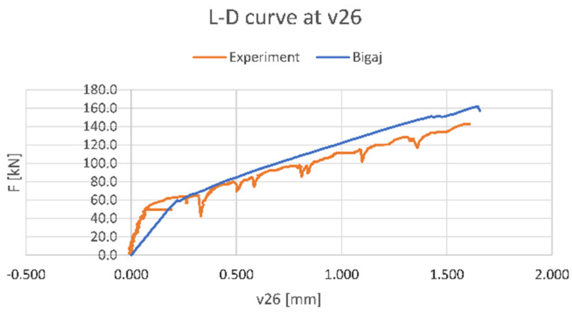

Fig. 8: Load-deflection curve – experiment vs. results of modelling, deflection at monitoring point v26

2) CEB-FIP 1990 model code

Crack patterns are compared with experiment in Fig. 9

The load-deflection curves of CEB-FIP 1990 model code are shown in Fig. 10 and Fig. 11. Material parameters were adopted from experiment research. Values that were achieved at maximal load force are:

ATENA: maximum load 149.30 kN; deflection at point v25 1.69 mm; deflection at point v26 1.52 mm.

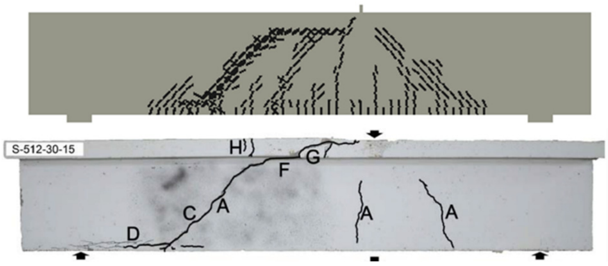

Fig. 9: Crack patterns from experiment and modelling (CEB-FIP).

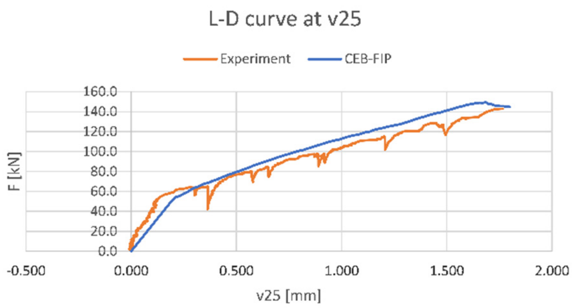

Fig. 10: Load-deflection curves at monitoring point v25.

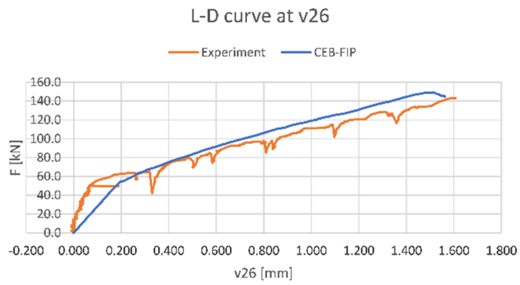

Fig. 11: Load-deflection curves at monitoring point v26.

5.2. Beam reinforced by GFRP

The bond-slip relationship plays such a significant role in beam reinforced by GFRP that it cannot be neglected. Assumption that bond model of concrete-GFRP is 2x smaller than in bond model of steel-concrete was considered. That means that we let ATENA computed bond models for steel but with input parameters we had (GFRP was modelled as steel). Then we manually reduced computed data in material section to half (50 %).

1) Bond by Bigaj

Crack patterns are compared with experiment in Fig. 12.

The load-deflection curve of Bigaj is shown in Fig. 13 and Fig. 14. Material parameters were adopted from experiment research. Values that were achieved at maximal load force are:

Experiment: maximum load 88.1 kN; deflection at monitoring point v25 4.608 mm; deflection at monitoring point v26 3.665 mm.

ATENA: maximum load 102.09 kN; deflection at point v25 4.91 mm; deflection at point v26 3.80 mm.

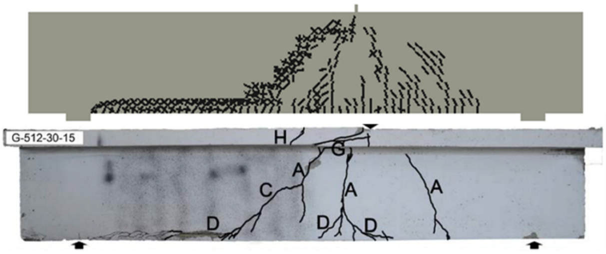

Fig. 12: Crack patterns from experiment and modelling (Bigaj).

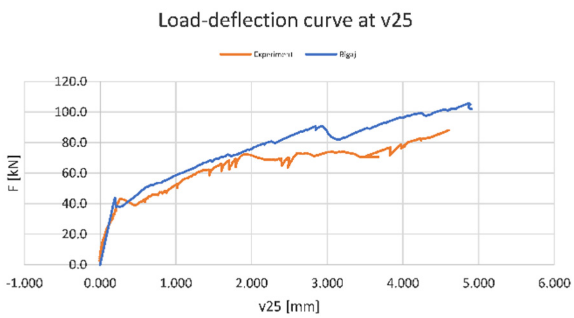

Fig. 13: Load-deflection curves at monitoring point v25.

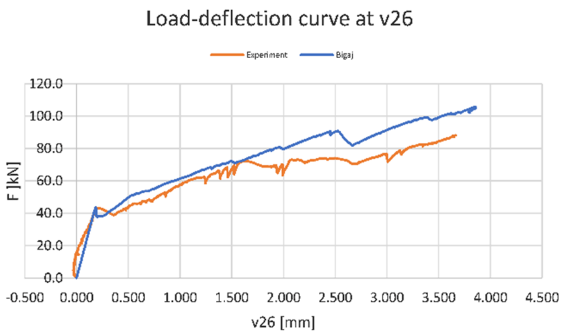

Fig. 14: Load-deflection curves at monitoring point v26.

2) CEB-FIP 1990 model code

Crack patterns are compared with experiment in Fig. 15.

The load-deflection curves of CEB-FIP 1990 model code are shown in Fig. 16 and Fig. 17. Material parameters were adopted from experiment research. Values that were achieved at maximal load force are:

ATENA: maximum load 98.40 kN; deflection of monitor point v25 4.96 mm; deflection of monitor point v26 3.78 mm.

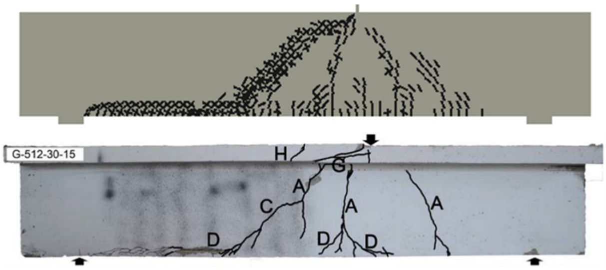

Fig. 15: Crack patterns from experiment and modelling (CEB-FIP).

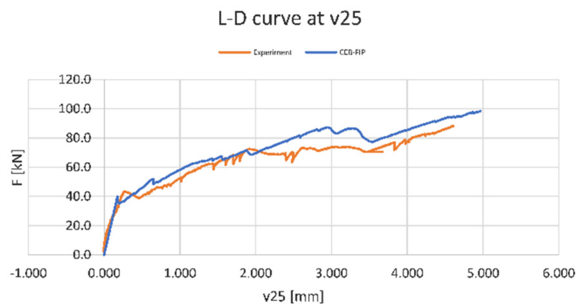

Fig. 16: Load-deflection curves at monitoring point v25.

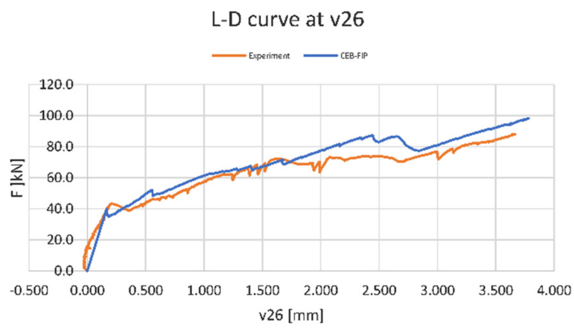

Fig. 17: Load-deflection curves at monitoring point v26.

6. Conclusion

Deterministic 2D nonlinear fracture mechanics computational models have been developed for both steel and GFRP reinforcement. Good agreement with experimental results has been achieved. Modelling of bond had to be included, it was a crucial for GFRP reinforcement. As these deterministic calculations represent the attempt to capture experiments at the level of one realization from fundamental set from statistical point of view, the next research will be focused on probabilistic modelling, considering uncertainties of material parameters. Consequently, design shear strength will be determined based on fully probabilistic and semi-probabilistic approaches.

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About Authors

Jakub VEČEŘE was born in Litomysl, Czech Republic. He is student of the Brno University of Technology in 4th grade (2023/2024).

Drahomír NOVÁK was born in Prostejov, Czech Republic. He received his M.Sc. from Brno University of Technology in 1984 and his PhD in 1991. Since 2003 he has been engaged as a full professor at Brno University of Technology and serves as head of Institute of Structural Mechanics. His research interests include reliability of concrete structures, stochastic fracture mechanics, semi-probabilistic methods and uncertainty quantification.

Renata KOTYNIA

1999 – PhD. Eng. degree on the topic "Deformability and load-bearing capacity of reinforced concrete members strengthened with carbon fiber laminates". 2012 – DSc. Eng. degree from Council Faculty of Civil Engineering, Architecture and Environmental Engineering, LUT on topic "Shear strengthening of reinforced concrete beams using polymer composites". 2020 – Prof. degree "FRP Composites for Flexural Strengthening of Concrete Structures. Theory, Testing, Design". 2012 - Associate Professor, FCEAEE, LUT; 2012 until now - Chair of the Scientific – Industry Council in FCEAEE, LUT; 2012 till now - Vice-Dean for Development, FCEAEE, LUT. 2016 - till now - Chair of Science Commission of Polish Association of Engineers and Civil Engineering Technicians 2019 till now – Head of Department of Concrete Structures, FCEAEE, LUT.

Damian SZCZECH

Responsible for the experimental tests and analytical modelling on shear in concrete beams reinforced with steel and FRP bars. Number of publications: in Publons 8, number of citations: 72; h-index = 4.

Monika KASZUBSKA

PhD Thesis "Analysis of the flexural reinforcement on the shear strength of the concrete beams without transverse reinforcement" - 2018. Responsible for the experimental tests and analytical modelling on shear in concrete beams. Number of publications: in Publons 17, number of citations: 139; h-index = 5 (WoS); h-index = 6 (Scopus).